Table 4.5.5 Estimated Time to Rehabilitate St. Mary River And Halls Coulee Siphons

Task	Duration
1) Replacement Bridge at St. Mary River	Prerequisite
2) Slope Stability Analyses	12-18 months
3) Feasibility Studies, Both Sites	6 months
4) Final Designs, Per Site	6 months
5) Construction Phases, Per Site	18-24 months
TOTAL TIME	42-54 months

4.6 HYDRAULIC DROPS

4.6.1 Structure Overview

The St. Mary canal empties into the North Fork of the Milk River after passing through five reinforced concrete drop structures. The total drop created by these structures is approximately 218 feet. The drop structures were originally designed by the BOR and construction was completed in 1915. The structures are similar in longitudinal and transverse section but vary in length and overall drop. The structures are numbered 1 to 5, from upstream to downstream.

Throughout the years, various concrete repairs have been made to the drop structures. These repairs have ranged from grouting of cracks in the slabs and side walls to replacement of entire sections of a structure due to concrete deterioration and failure. Maintenance of these structures has been a regular practice over the years and to date is an ongoing process. A recent failure within Drop No. 2 resulted in replacement of an entire chute and side wall section within that structure.

4.6.2 Existing Conditions and Deficiencies

An initial cursory inspection of the canal and drop structures was performed by the project team on October 13, 2004. Each of the five drop structures were inspected in further detail during a site visit on November 10, 2004. The system was not in operation at the time of the inspections. However, the plunge pools were inundated, which prevented a complete inspection of the plunge pool slab and lower walls.

Personnel from the BOR performed a detailed inspection of the drops in October of 1999. Water was pumped out of Drops 2, 3, and 4 so that the structures could be inspected in a dewatered condition. The results of that inspection are documented in the Saint Mary Canal O&M Condition Assessment Trip Reports (January, 2000).

The following paragraphs present a brief discussion of each drop structure and observations from our recent inspections.

<u>Drop Structure No. 1</u>

Drop No. 1 has a total length of approximately 215 feet and a vertical drop of approximately 36.5 feet. An overall view of the structure is shown in Figure 4.6.1. This structure appears to be in the best condition of all five drops. The spillway chute downstream of the V-notch has experienced moderate concrete spalling. However, the approach slab has experienced moderate to severe spalling as shown in Figure 4.6.2.

Figure 4.6.1 Looking west towards Drop No. 1 (11/10/04).

Figure 4.6.2 Looking at approach section of Drop No. 1. Note condition of concrete (11/10/04).

Water in the plunge pool prevented a complete inspection of the condition of the lower walls or slab. Rebar is exposed on the visible portion of the vertical face just downstream of the chute as shown in Figure 4.6.3. There is also a large crack, with exposed rebar, at the left wall to chute slab interface just upstream of the vertical drop as shown in Figure 4.6.4.

Figure 4.6.3 Looking at vertical face downstream of chute section of Drop No. 1 (11/10/04).

Figure 4.6.4 Looking at left (north) retaining wall at terminus of chute section (Drop No. 1). Note structural cracking (11/10/04).

The condition of the foundation under the entire structure is unknown as is the case with all of the structures except as discussed for Drop No. 3.

Drop Structure No. 2

Drop No. 2 has a total length of approximately 205 feet and a vertical drop of approximately 29.5 feet. An overall view of the structure is shown in Figure 4.6.5. A section of this structure (slab and sloping side walls) was replaced after a partial failure that occurred in 2002 as shown in Figure 4.6.6.

Figure 4.6.5 Downstream view of Drop No. 2. Note stilling basin landslide in background (11/10/04).

Figure 4.6.6 Downstream view of chute of Drop No. 2. Note new section replaced in 2002 and extent of landslide in background (11/10/04).

The ogee crest appears to be in good condition as shown in Figure 4.6.7. The slab upstream and downstream of the replaced section shows widespread spalling. The chute slab is shown in Figure 4.6.8, upstream and downstream of the replaced section, respectively. The condition of the foundation under the entire structure is unknown.

Figure 4.6.7 View of ogee crest of Drop No. 2 (11/10/04).

Figure 4.6.8 Downstream view of chute section of Drop No. 2 (11/10/04).

A section of the left wingwall (perpendicular to end of the plunge pool) appears to be failing as shown in Figure 4.6.9. Rebar is exposed on the downstream face of this wall and a seep hole has developed along the opposite side of this wall as shown in Figures 4.6.10 and 4.6.11, respectively. Sink holes are an indication of the loss of backfill soil due to seepage. Also, severe deterioration of the concrete has occurred on the vertical face of the plunge pool with rebar exposed as shown in Figure 4.6.12. Water in the plunge pool prevented a complete inspection of the condition of the lower walls or slab. It appears that the downstream section of the chute floor, just before the vertical drop, is possibly settling as evidenced by the type of cracking and opening of joints within the chute section also shown in Figure 4.6.12.

Figure 4.6.9 Left (north) wingwall of Drop No. 2 is failing (11/10/04). Also, see Figures 4.6.10 and 4.6.11.

Figure 4.6.10 North wingwall of Drop No. 2. Note wall displacement, concrete loss and exposed reinforcement (11/10/04).

Figure 4.6.11 Sink hole developing behind north wingwall of Drop No. 2 (11/10/04).

Figure 4.6.12 Looking at terminus of chute section for Drop No. 2. Note condition of plunge pool headwall and end of chute (11/10/04).

Drop Structure No. 3

Drop No. 3 has a total length of approximately 139.5 feet and a vertical drop of approximately 27.8 feet. An overall view of the structure is shown in Figure 4.6.13. At the time of the inspection, a BOR maintenance crew was in the process of replacing the entire chute slab from the first joint downstream of the ogee crest to the end of the chute. This work is shown in Figures 4.6.13 and 4.6.14. In talking with the crew, upon removal of the slab, there was no evidence of piping or voids observed in the foundation materials below the chute slab. The foundation was noted to consist of large cobblestones with drains along the full length of the chute at the slab to chute wall interface.

Figure 4.6.13 Looking upstream toward Drop No. 3 (11/10/04).

Figure 4.6.14 Photo shows maintenance work being performed on Drop No. 3 (11/10/04).

The crew pointed out that they had recently filled a large sinkhole on the backside of the north (left) training wall just downstream of the end of the chute. This sinkhole extended the full height of the training wall. At the base of the wall at the location of the sinkhole, the concrete deterioration has resulted in a hole all the way through the wall. Figure 4.6.15 shows this deteriorated area and the fill placed by BOR crews. The sinkhole formed due to moisture seepage which caused the soil to pipe or “wash” through the hole in the concrete.

Figure 4.6.15 Looking at north (left) training wall in plunge pool of Drop No. 2. BOR crews filled large sink hole behind wall (11/10/04).

The approach and ogee crest section appear to be in good condition as shown in Figure 4.6.16. The concrete is deteriorated with exposed rebar on the vertical face of the plunge pool as shown in Figure 4.6.17. Water in the plunge pool prevented a complete inspection of the condition of the lower walls or slab. Extensive concrete deterioration has also taken place near the base of the retaining wall on the right side of the plunge pool as shown in Figure 4.6.18. As has already happened at the base of the left wall and as discussed in the previous paragraph, it may only be a matter of time before the concrete deterioration extends through the thickness of the right training wall resulting in similar piping and subsequent sinkholes on the backside of these walls.

Figure 4.6.16 Ogee section of Drop No. 3 (11/10/04).

Figure 4.6.17 Condition of plunge pool headwall at drop No. 3 (11/10/04).

Figure 4.6.18 Typical condition of south (right) training wall at Drop No. 3 (11/10/04).

Drop Structure No. 4

Drop No. 4 has a total length of approximately 340 feet and a vertical drop of approximately 67 feet. An overall view of the structure is shown in Figure 4.6.19. The chute is in poor to marginal condition as shown in Figure 4.6.20. There are several areas within the chute that have severe deterioration. For example, the deterioration in the chute slab (approximately 4 inches wide by 6 inches long) shown in Figure 4.6.21. and in the chute side wall slab (approximately 28 inches wide by 32 inches long) shown on the left side of Figure 4.6.22 is of concern. These deteriorated areas vary from partial depth to full depth of the concrete and could result in piping under the slab or complete blowout of the slab concrete if not corrected soon. Widespread spalling of the concrete is evident near the downstream end of the chute as shown in Figure 4.6.23.

Figure 4.6.19 Looking downstream at Drop No. 4 (11/10/04).

Figure 4.6.20 Looking at chute section of Drop No. 4 (11/10/04).

4.6.21 Close-up of concrete deterioration of Drop No. 4 chute floor (11/10/04).

As is the case with the other drop structures, concrete deterioration and exposed rebar is visible on the vertical headwall of the plunge pool. Water in the plunge pool prevented a complete inspection of the condition of the lower walls or slab.

Figure 4.6.22 Looking upstream at Drop No. 4. Note concrete deterioration (11/10/04).

Figure 4.6.23 Looking at terminus of chute section (Drop No. 4). Note the degree of concrete spalling (11/10/04).

Drop Structure No. 5

Drop No. 5 has a total length of approximately 259 feet and a vertical drop of approximately 57.3 feet. Past minor repairs to the chute and side walls are evident throughout the structure. Moderate to severe concrete spalling and cracking exists throughout the structure and the spalling is heaviest near the bottom of the slab as shown in Figure 4.6.24. The depth of the concrete deterioration in this section is between one to three inches.

Figure 4.6.24 Heavy spalling of chute floor in Drop No. 5 (11/10/04).

A couple of areas of severely deteriorated concrete in the chute slab are a concern. One of the areas is shown in Figure 4.6.25. This particular hole appears to be through the thickness of the slab and would likely lead to piping or complete blowout of the slab concrete if not corrected soon.

Figure 4.6.25 Hole in chute floor of Drop No. 5 (10/13/04).

The training walls on both sides of the plunge pool are heavily eroded and damaged as shown in Figure 4.6.26. Exposed rebar beyond the top of both walls can be seen in the photo. The plunge pool water level was high during the inspection, so it was not possible to inspect the condition of the lower walls or slab. Exposed rebar and concrete deterioration is evident towards the top of the vertical face of the plunge pool as shown in Figure 4.6.27 around both drains.

Figure 4.6.26 Heavily damaged/eroded wing and training walls on Drop No. 5 11/10/04).

Figure 4.6.27 Terminus of chute section of Drop No. 5. Note condition of plunge pool headwall (11/10/04).

General

Past performance of the drop structures has shown that during high flows, water “jumps” out of the chutes and onto the side banks towards the bottom of the structures. If allowed to continue, this will result in side bank erosion and undermining of the structures and eventual failure.

During a recent meeting with BOR personnel on December 9, 2004, the problem of snow buildup upstream of the drop structures during initial spring startup was discussed. It was noted by BOR personnel that during initial filling of the system, the flowing water picks up the snow and ice and transfers it downstream. The suspended ice collects within the transition to the drop. Clearing of this accumulation of ice and snow is routinely required.

Hydropower Studies

According to the Regional Feasibility Study of North Central Montana [Reclamation 2004], hydropower development has previously been investigated at the St. Mary Canal terminal drop structures. A private enterprise evaluated a small hydropower facility at the St. Mary Canal drops in the 1980s. Apparently, economic factors precluded hydropower development at that time. Documentation of this study was not available for review.

A low-head hydroelectric evaluation and inventory completed under Public Law 95-482 [Reclamation 1980] included an individual assessment of each of the five drops on the St. Mary Canal. The study assumed replacement of each drop with a penstock and small hydroelectric facility. During the first round of the evaluation, field costs for site-specific features such as penstocks, tailrace, switchyard equipment, transmission lines, and other costs were estimated on a uniform basis. A summary of the first round of the evaluation is shown in the table below.

Table 4.6.1 Summary of Low-Head Hydroelectric Evaluation Performed by BOR in 1980

Assessment of Small Hydroelectric Development at Existing Facilities Round One Evaluation Summary [Reclamation 1980]					
Drop	**Ave. Head (feet)**	**Installed Capacity (kilowatt)**	**Ave. Annual Energy (GWH)**	**Investment Cost ($1000)**	**Benefit-Cost Ratio**
1	36	906	2.93	2328	0.63
2	29	664	2.18	2185	0.51
3	27	556	1.87	2104	0.45
4	61	1939	5.85	2643	1.08
5	51	1616	4.88	2617	0.92

Four of the sites were eliminated at the end of the first round of the evaluation due to a benefit-cost ratio less than one. During the second round of the evaluation, layout sketches of the possible power plant location and equipment were prepared and the costs were upgraded to an appraisal level to reflect site specific conditions. As a result, the investment cost for installation of a small hydroelectric facility at Drop 4 almost doubled, which reduced the benefit-cost ratio to less than one and thus eliminated the site from further consideration.

More recently, at least one company indicated interest in hydropower development at the St. Mary Canal drops. The Federal Energy Regulatory Commission (FERC) issued a preliminary permit on October 22, 2001 to BAE Energy in Cut Bank, Montana to study development of a small hydroelectric facility at the St. Mary Canal drops. The preliminary permit was surrendered on July 26, 2002. The request for termination of the preliminary permit indicated that economic conditions were unfavorable.

The BAE Energy preliminary permit application for hydropower development at the St. Mary Canal drops proposed to replace the existing drop structures with a new canal approximately 1.5 miles long and a 9.5 foot diameter penstock approximately 1,300 feet long. The proposed penstock would supply two 1.4 MW Francis turbines. The proposed average flow was 500 cubic feet per second with an average head of 98 feet. Average annual generation was proposed to be approximately 21,000 MW-hours. Approximately three miles of new 12.5 KV three-phase transmission line would be required to connect to the existing grid. The study was terminated before any of these values were confirmed.

4.6.3 Rehabilitation Alternatives

Feasibility level costs were developed in the Engineering Appendix Report (April, 2003) for replacement of the five drop structures. Costs were developed for four flow capacities; 500 cfs, 670 cfs (estimated current flow in canal), 850 cfs (original design flow), and 1000 cfs. For each of the four flow capacities, costs were developed for three structure concepts; baffled apron drop, pipe drop, and chute with a stilling basin (similar to existing).

The BOR’s recommended alternative was a pipe drop for all five structures. The cost for this alternative either fell in between the baffled apron drop and the chute and stilling basin or was below the cost of both of the other alternatives for all five drops for all flow capacities. The chute and stilling basin was the most expensive alternative for all five drops and all flow capacities. Several advantages were listed for the pipe drop over the other two alternatives, including: access across the canal, elimination of safety hazards associated with open structures, and elimination of O&M costs associated with snow and ice removal required for early spring use.

In our opinion, the open chute has more advantages than a pipe drop which deserves additional consideration during the Feasibility Study. Pipe drops are a closed conduit, and as such, have a limited capacity and are prone to more O&M issues related to icing, floating debris and blockage. In our opinion, the current issue with snow and ice in the canal impacting the chutes is related to the controlling ogee entrance section and insufficient canal freeboard to account for the development of backwater. The chutes can be equipped with access platforms for personnel or vehicles to cross the canal. Also, it is the experience of several members of our team that open chutes are more cost effective than a pipe drop. Also, in our opinion, there may be opportunity to combine 2 or more drops into a single drop. This may or may not reduce costs but should be evaluated.

Since the decision of whether or not to develop the hydroelectric power will impact the types and locations of replacement drop, the feasibility of hydropower should be updated before the drops are designed. Considerations should include the Blackfeet Nation's plans to develop wind farms east of Duck Lake.

4.6.4 Estimated Rehabilitation Costs

The cost estimates presented by the BOR are dated on March 21, 2003. These cost estimates were reviewed. Discrepancies observed were noted between the summary table on page 3 and the individual cost estimate tables for separate drops (BOR, 2003). The BOR has indicated that the summary table values are correct. The cost estimating worksheets did not specifically add 5% for Tribal fees. Assuming construction would occur in the summer of 2007, it is appropriate to update these estimates by escalating the costs by 3% per year for four years (x 1.1255). The following tables present the cost estimates originally prepared by the BOR for the pipe drop alternative and those projected to 2007 with 5% Tribal fees.

Table 4.6.2
Estimated Costs to Rehabilitate Drop Structure No. 1

Canal Capacity (cfs)	BOR Cost Estimates - 2003			Projected Costs – 2007(1)
	Baffled Apron Drop	Pipe Drop	Chute & Stilling Basin	Pipe Drop
500	$620,000	$590,000	$840,000	$698,000
670	$660,000	$620,000	$950,000	$733,000
850	$740,000	$810,000	$960,000	$957,100
1000	$860,000	$840,000	$1,100,000	$992,700

(1) = [BOR cost) * 1.1255] * 1.05

Table 4.6.3
Estimated Costs to Rehabilitate Drop Structure No. 2

Canal Capacity (cfs)	BOR Cost Estimates - 2003			Projected Costs - 2007[(1)]
	Baffled Apron Drop	Pipe Drop	Chute & Stilling Basin	Pipe Drop
500	$660,000	$730,000	$930,000	$863,000
670	$730,000	$730,000	$1,000,000	$863,000
850	$770,000	$890,000	$1,050,000	$1,051,800
1000	$890,000	$900,000	$1,200,000	$1,063,600

[(1)] = [BOR cost) * 1.1255] * 1.05

Table 4.6.4
Estimated Costs to Rehabilitate Drop Structure No. 3

Canal Capacity (cfs)	BOR Cost Estimates - 2003			Projected Costs - 2007[(1)]
	Baffled Apron Drop	Pipe Drop	Chute & Stilling Basin	Pipe Drop
500	$530,000	$630,000	$860,000	$745,000
670	$600,000	$660,000	$890,000	$780,000
850	$590,000	$790,000	$1,000,000	$933,600
1000	$750,000	$810,000	$1,100,000	$957,200

[(1)] = [BOR cost) * 1.1255] * 1.05

Table 4.6.5
Estimated Costs to Rehabilitate Drop Structure No. 4

Canal Capacity (cfs)	BOR Cost Estimates - 2003			Projected Costs - 2007[(1)]
	Baffled Apron Drop	Pipe Drop	Chute & Stilling Basin	Pipe Drop
500	$820,000	$810,000	$1,050,000	$958,000
670	$970,000	$840,000	$1,100,000	$993,000
850	$1,100,000	$1,050,000	$1,125,000	$1,240,900
1000	$1,250,000	$1,100,000	$1,350,000	$1,300,000

[(1)] = [BOR cost) * 1.1255] * 1.05

Table 4.6.6
Estimated Costs to Rehabilitate Drop Structure No. 5

Canal Capacity (cfs)	BOR Cost Estimates - 2003			Projected Costs - 2007[1]
	Baffled Apron Drop	Pipe Drop	Chute & Stilling Basin	Pipe Drop
500	$840,000	$690,000	$1,100,000	$816,000
670	$950,000	$700,000	$1,200,000	$828,000
850	$1,000,000	$890,000	$1,300,000	$1,051,800
1000	$1,100,000	$930,000	$1,450,000	$1,099,100

(1) = [BOR cost) * 1.1255] * 1.05

4.6.5 Rehabilitation Schedule

Previous reports have presented various alternatives for rehabilitation. The majority of the recommendations call for complete replacement of the five drop structures due to their overall deteriorated condition and age. Sections of the chute slab and side walls within areas of severe concrete deterioration could fail at any time. In addition, an increase in piping and subsequent sink holes is a strong possibility along the downstream training walls on either side of the plunge pool. The potential for piping directly under the chute slab just upstream of the plunge pool is also a strong possibility due to the severe deterioration in the vertical headwall at the end of the chute.

Drops 4 and 5 represent the worst condition relative to Drops 1 and 2. Portions of Drop 3 are being restructured during the off-season of 2004-2005. Due to the potential failure of the drop structures at any time, a top priority during the next phase of work should be to evaluate the alternatives and select an approach for rehabilitation or replacement. The feasibility of hydropower needs to be determined initially, as this would impact the rehabilitation on the drops.

Replacement of all five drop structures could be completed within 24 months. The construction can be accomplished during the normal diversion season.

Table 4.6.7 Estimated Time to Rehabilitate Hydraulic Drops No. 1 Through No. 5

Task	Duration
1) Hydropower Feasibility Study	4 months
2) Replacement Feasibility Study	4 months
3) Final Design *	8 months
4) Construction Phase	24 months
TOTAL TIME	40 months

* Does not include costs for design of hydropower machinery.

4.7 CANAL PRISMS

4.7.1 Structure Overview

The St. Mary Canal was construction between 1907 and 1915, and its original design capacity is 850 cfs. The canal is approximately 29 miles long and is an earthen, unlined, one-bank, contour canal. The original prism had the following parameters.

- 26-foot flat bottom trapezoidal section
- 2:1 (H:V) side slope fill sections
- 1½:1 side slope in cut sections
- invert slope of 0.00010 feet per foot (0.53 feet per mile)
- constructed of natural materials

The canal has been realigned and relocated in several locations since original construction. A significant relocation involved abandoning an elevated flume and placing the flow in a replacement canal between the outlet of St. Mary River Siphon and Spider Lake. Other relocations have been minor but warranted due to slope instabilities.

Cross drainage consisting of culvert structures under the prism exist at seven locations. All other drainages flow directly into the canal and are term stormwater inflow. Grassed overflow sections were constructed at several locations to accommodate excess inflows. The cross drains are listed below.